The Collapse and Reconstruction of a Prestressed Concrete Box-Section Girder

Ecroulement et assainissement d'une poutre en caisson précontrainte

Einsturz und Sanierung eines Hohlkastenträgers aus Spannbeton

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SUMMARY

During the construction of a prefabricated hall roof, a 25 m long prestressed box-section girder collapsed. The major cause of the failure was an inadequate structural design of the support regions. There was no transverse end beam present and support bearing points were not placed beneath the webs of the girder. The resulting transverse bending together with insufficient end anchorage led to the collapse. 45 similar beams, of which half were already erected, were in consequence locally strengthened in the support regions. Thus an original remedial solution, scarcely externally visible, was developed and tested through an on-site load test.

RÉSUMÉ

Lors du montage du toit préfabriqué d'un hangar, une poutre en caisson précontrainte de 25 m s'écroula. L'origine principale du dégât résidait en une conception inadéquate de la poutre dans la région d'appui. Il n'existait pas d'entretoise à l'extrémité et les points d'appui ne se trouvaient pas sous les âmes. La flexion transversale qui s'en suivit, combinée avec une sécurité d'appui insuffisante, conduisit à l'écroulement. 45 poutres analogues, dont la moitié était déjà en place, durent par la suite être consolidées dans la zone d'appui. Une solution d'assainissement originale fut développée dans ce but; elle est à peine visible de l'extérieur, et fut contrôlée par des essais sur le chantier.

ZUSAMMENFASSUNG

Während der Montage eines vorfabrizierten Hallendaches stürzte ein 25 m langer vorgespannter Hohlkastenträger ein. Die Hauptgründe für das Versagen waren die ungenügende Ausbildung des Trägers im Auflagerbereich, d.h. es war kein Endquerträger vorhanden, und die Auflagerpunkte waren nicht unter den Stegen angeordnet. Die dadurch bewirkte Querbiegung führte zusammen mit einer ungenügenden Auflagersicherung zum Einsturz. Sämtliche 45 analogen Träger, die zur Hälfte bereits montiert waren, mussten nachträglich im Auflagerbereich verstärkt werden. Dabei wurde eine originelle, von aussen kaum sichtbare Sanierungslösung entwickelt und durch einen Grossversuch auf der Baustelle überprüft.

1. COLLAPSE DURING ERECTION

On the 25th September 1974 a 20 x 25 m section of a prefabricated hall roof col-
lapsed during erection. Within this collapse a box-section girder spanning 25 m
and a series of immediately placed 10 m long slab units fell from a height of
around 5 m onto the hall floor and were extensively damaged.

This surprising event occurred at the building site of the Swiss National Rail-
way's new rapid goods station, Berne. The new rapid goods concept of the Swiss
National Railway provides for the turn over of goods traffic on select trains to
be concentrated in three centres (Berne, Zürich, Dänikon).

The Berne rapid goods station consists mainly of the goods turnover hall, which
partially has a basement, and a five storey tower with offices and service rooms.
The ground level for the hall (railway line trenches and platforms) together with
the columns are formed from reinforced concrete. The roof was prefabricated and
consists of ribbed slabs (L = 90 m, H = 25 cm) that are supported by trapezoidal
box-section girders (L = 14.8 - 25.6. m, H = 1.4 m). The roof finish is formed
with a thin layer of waterproofing material glued to the concrete. No thermic
insulation was required. A plan and characteristic sections are shown in Fig. 1.
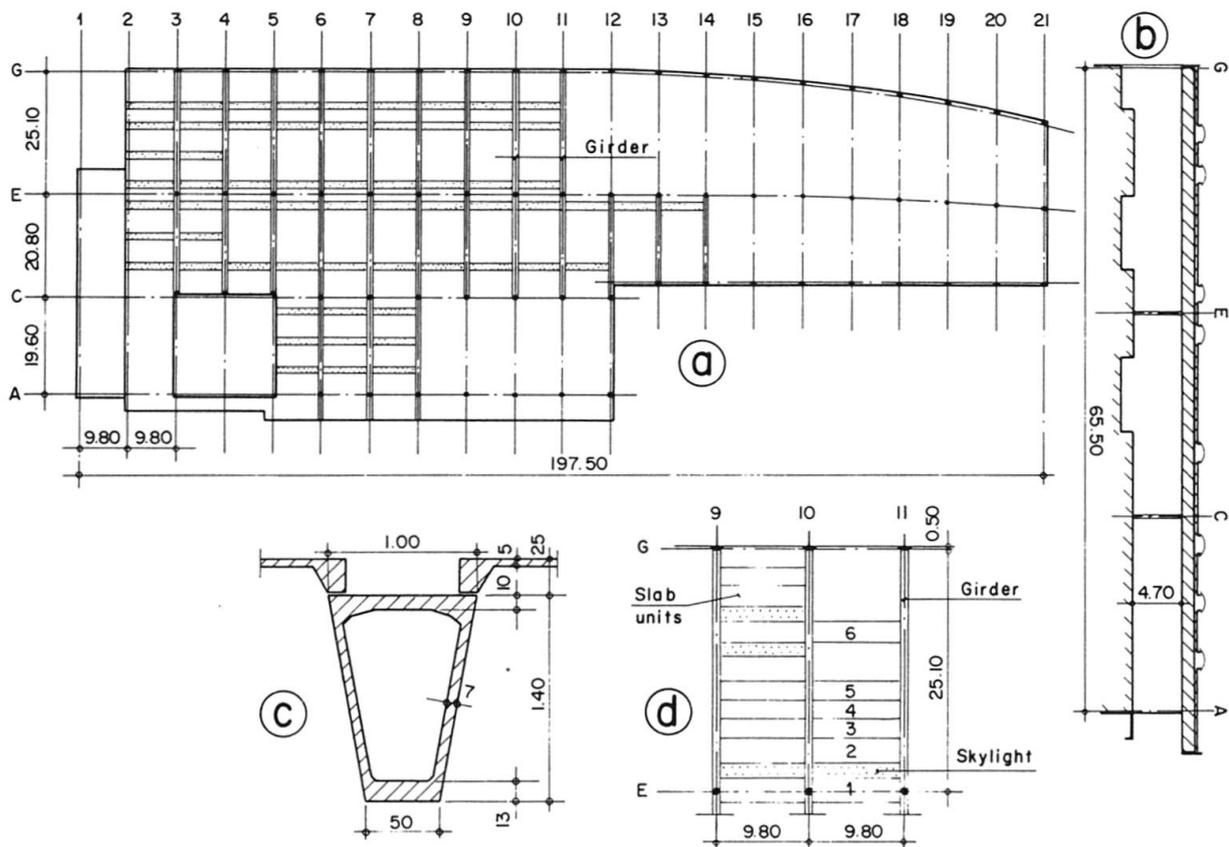
Fig. 1   Rapid goods station Berne

        a) Hall floor plan (position of construction at the time of collapse)
        b) Hall section on grid line 8
        c) Girder cross-section
        d) Plan of the collapse section

The box-section girders are simple beams with constant cross-section. The transverse end beam was eliminated to achieve an expedient fabrication and a good accessibility. The accessibility was desired so that conduits could be run through the girders, overhead train line fixings could be checked etc.

The collapse occurred after about half the hall roof was erected. The day before the collapse, the girder 10/E-G and the slabs in section 9-10/E-G had been placed. The next day the erection of the girder 11/E-G and a large part of the slabs in section 10-11/E-G was carried out. Several minutes after the placing of slab unit No. 6 the girder 10/E-G failed. This was without additional external loading (see Fig. 1).

2. SITUATION AFTER THE COLLAPSE

The situation on the construction site immediately after the collapse is shown in Figs. 2, 3 and 4. The first examinations showed immediately that the girder 10/E-G had somehow broken through at the support at column E/10. From there it had fallen to the ground and in doing so also tore away from the support G/10.

One check made immediately showed that many of the already erected girders had dangerous cracks in the support regions. Therefore as a first measure, all girders were provided with provisional underpropping.
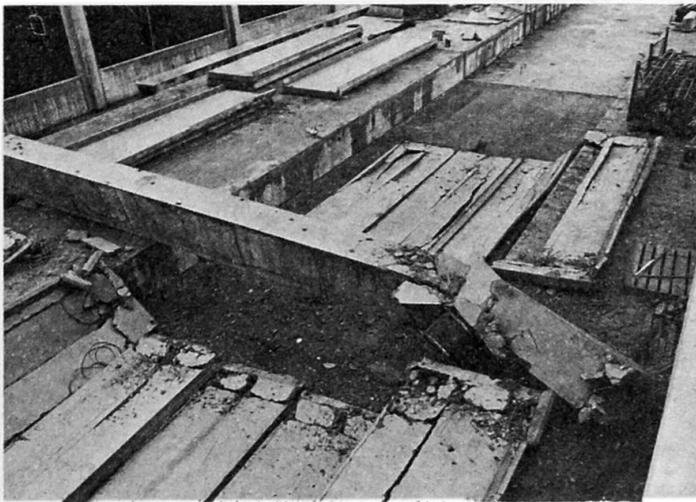

Fig. 2

View from E/9 towards the collapsed box-section girder. In front and behind are the collapsed roof slab units.

Fig. 3

View from E/11 towards the collapsed girder. The prestress wires of the bottom flange were fully exposed in the collapse.
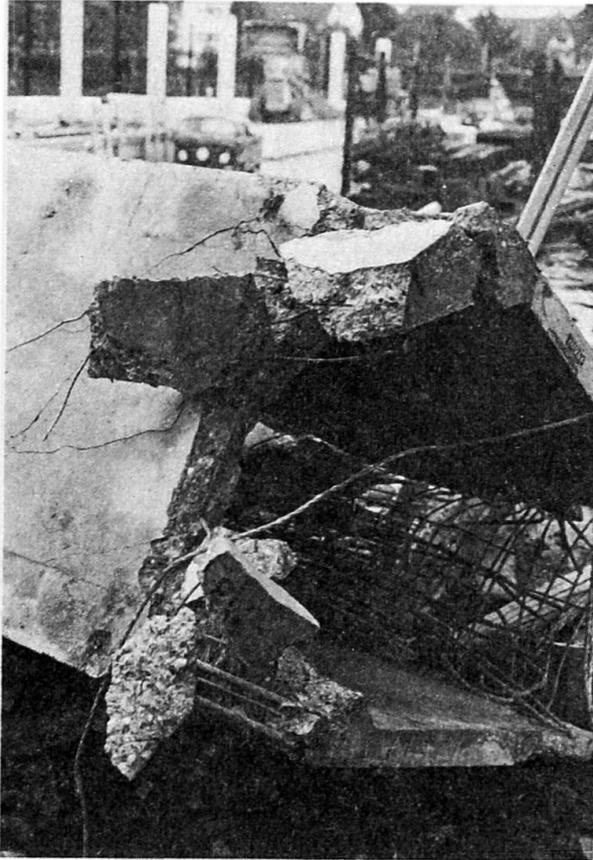
Fig. 4

View of girder end E/10. The girder
has broken through the bottom and
top flanges at the support.

3. CLARIFICATION OF COLLAPSE CAUSES

From the beginning it was suspected that the causes of the collapse lay with the
detailing required in the support region of the box section girders for local
stress actions.

3.1 Detail of the Support Region

The to scale dimensions and reinforcement of the support region are shown in
Figs. 5 and 6. The girder had no transverse end beam. In the bottom flange were
10 cm long x 5 cm wide holes for 3 cm Ø bolts cast into the column. The support
bearing points in the centre of these holes were to scale 10 cm from the girder

Fig. 5

Dimensions and reinforcement
of the girder support region.
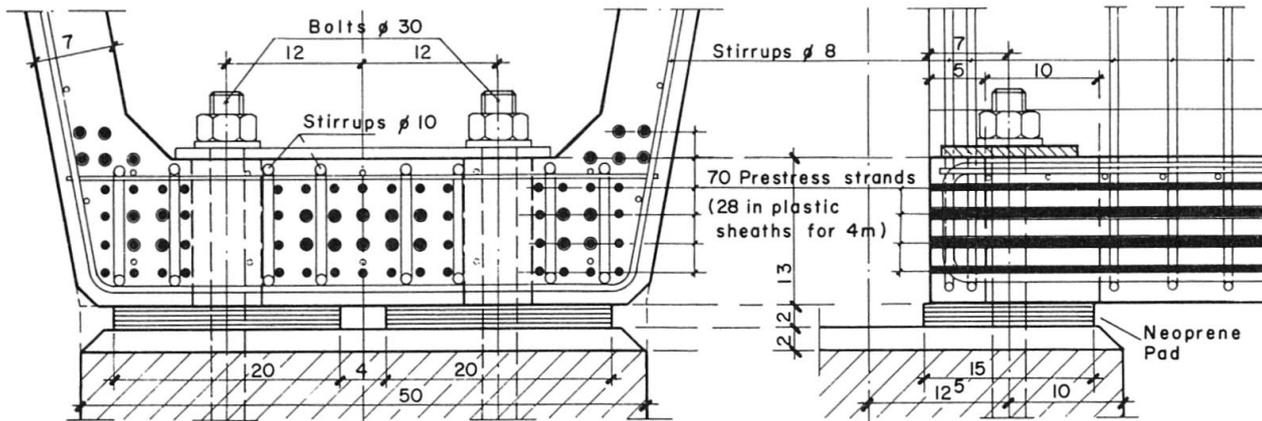
Fig. 6 Layout of the support region. Support in extreme situation.

end. However this value could be reduced to nearly 7 cm when measurement varia-
tions were taken into account. This was the case with the collapsed girder. The
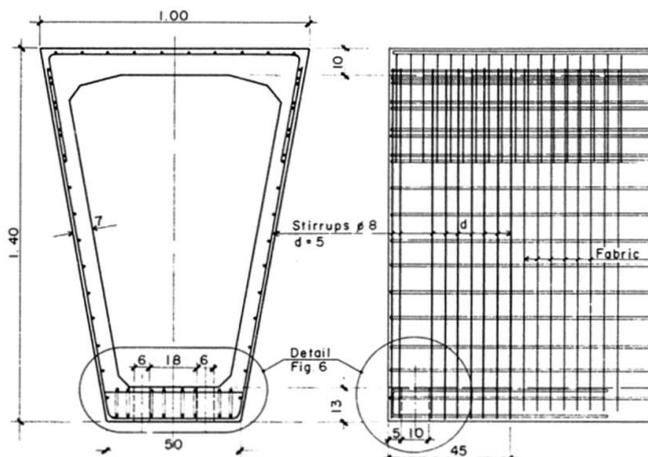workers on site informed that the bolts had scarcely fitted through the holes in
the girder. In the transverse direction the support points did not sit beneath
the webs, but were about 10 cm within the cross-section.

The actual support consisted of two neoprene layers 200 mm x 150 mm x 21 mm pro-
vided with centre holes. After erection of the girder a steel plate and nuts were
placed over the bolts. The arrangement is shown in Figs. 7 and 8.

Fig. 7

View from beneath towards the
column head E/10. Right is the
girder 10/C-E. Left was the
collapsed girder 10/E-G.

Fig. 8

View from above of the column
head E/10. Righthand end of the
beam 10/C-E with holes in the
bottom flange and removed nuts
and steel plate. Remainder of
lefthand end of the collapsed
girder 10/E-G with the still
present nuts and top steel plate.
Beneath the rubble are both the
neoprene pads.

3.2 Transverse Bending with Central Loading

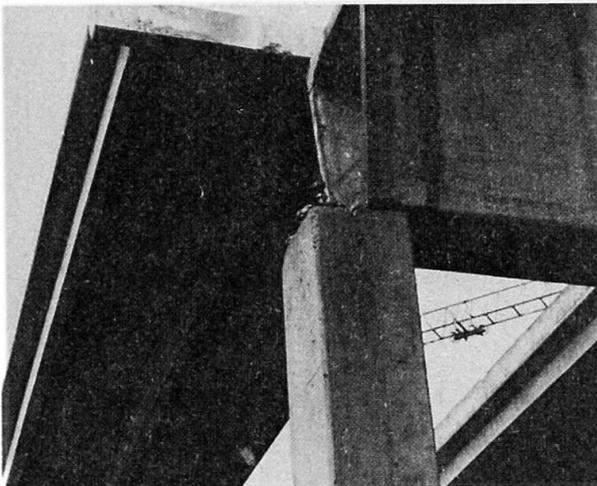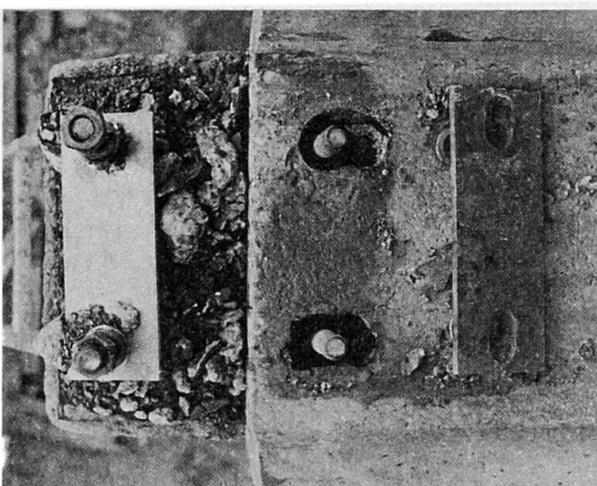The self weight of the box-section girder produces a support reaction of around 110 kN. With the placement of all slab units on both sides of the girder a further 220 kN results. This gives in total a 330 kN reaction for the load condition which strongly corresponds to that at collapse. The roof finishing layer and the snow load increase this reaction to 500 kN. Code SIA 162 [1] directs that a safety factor of 1.8 be applied. Consequently the allowable ultimate load is 900 kN. In contrast to this the support reaction at collapse was only about one third of this.

Firstly the transverse beinding should be estimated for a concentric loading of the basic structural system's dead weight i.e. for a support reaction of 330 kN. A beam element at the end of the box-section girder, shown in Figs. 9a and 9b, is considered for this purpose. A closed frame is formed which is acted upon by the shear flow of the shear force and by the support reactions. Since both support bearing points are not located beneath the webs, considerable transverse bending moments arise. These, for the assumption of an uncracked section, are shown in Fig. 9c. The moment in the bottom flange comes to 15.5 kNm. With the reinforcement present over a locally supported width of 30 cm the corresponding yielding moment comes to only 3.5 kNm. This means that over both the support bearing points a plastic hinge must be assumed to occur. When one calculates for the remainder with the assumption of an uncracked frame, then the moment diagram of Fig. 9d results. Here the existent yielding moment in the web of 7.5 kNm is considerably exceeded. Thus a possible equilibrium condition can likewise not be found. That the beam did not collapse earlier with a smaller load therefore requires some explanation:

Due to the formation of the plastic hinges at the support bearing points considerable displacements arise. These cause a certain shifting through the neoprene pads of the produced reaction forces towards the webs. Thus the transverse bending is significantly reduced.
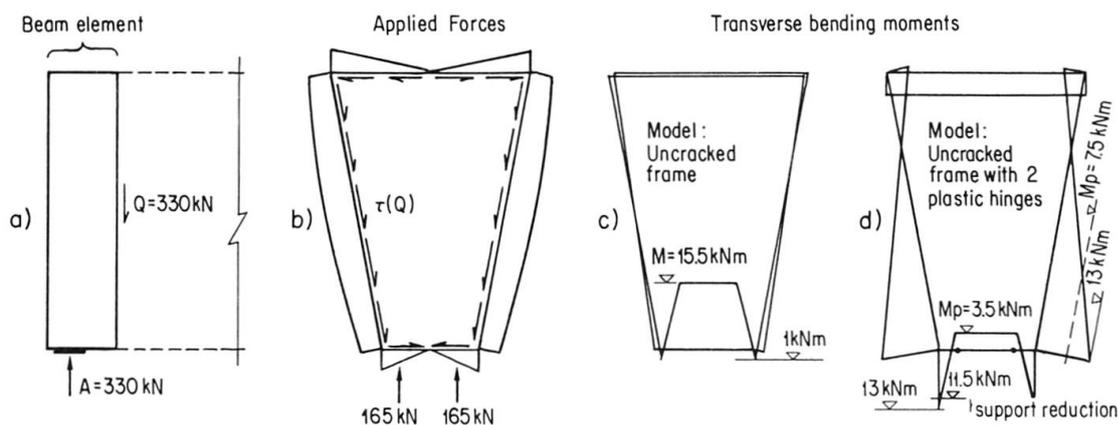
Fig. 9   Transverse bending of the support region after placement of the slab units on both sides (concentric loading).

3.3 Transverse Bending with Eccentric Loading

At the box-section girder supports there occurs during erection an eccentric loading condition and resulting torsion. This occurs when the roof slab units are placed first on one side only. Here a reaction force of 220 kN results with an additional torsional moment of 46 kNm. The stresses in the beam end element (see

Fig. 10a) are established through the superposition of the two load cases "concentric loading" and "torsion".

The concentric loading comes to 220/330 = 2/3 of the previously analysed loading. Each support reaction comes to +110 kN. The torsional loading consists of a constant shear flow τt and equal but opposite reactions of ± 190 kN. Fig. 10b gives for the sum of both load cases the shear flow and support reactions of 110 ± 190 = +300 kN or -80 kN.

By a similar procedure to that in the previous section for firstly an uncracked frame, the moment diagram shown in Fig. 10c results. This however can not occur since the existent yielding moments(resisting) are exceeded. After the introduction of plastic hinges over the right support bearing point and at the transition of the bottom flange to the left web, the moment diagram in Fig. 10d results. This is however likewise not possible. As described in the previous section, there is a possible sidewards displacement of the support reactions towards the webs. This is caused by plastic deformations and should be taken into consideration if an equilibrium condition is to be found.
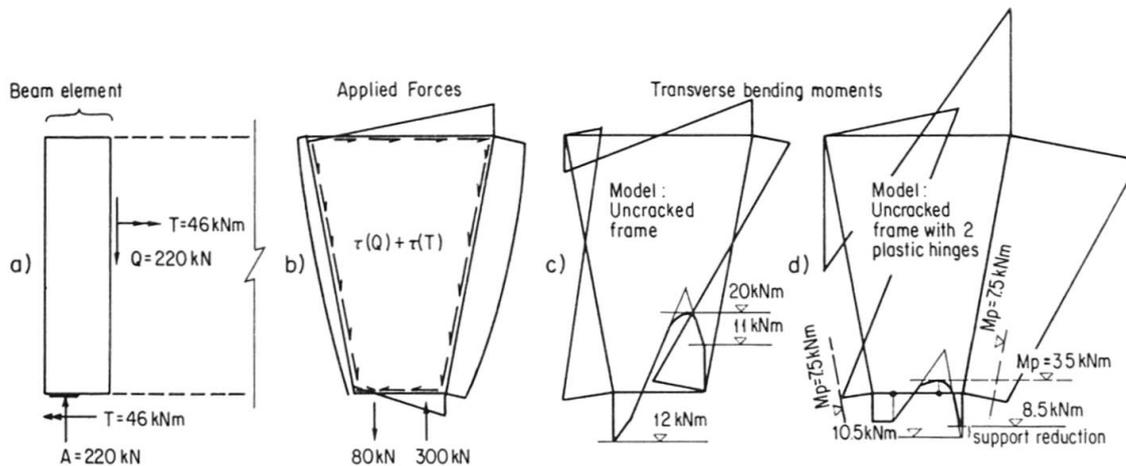

Fig. 10   Transverse bending of support region after placement of
          the slab units on one side only (eccentric loading).

3.4 End Anchorage

In Fig. 11a the force system in the web plane of the support region is schematically shown i.e. with a truss model that simplifies the actual situation. The inclined compressive force D stands in equilibrium with the tensile chord force Z and the support reaction A. If in addition there acts an external support force H, e.g. a secondary reaction, then Z must be increased with respect to H. If one takes an angle of inclination of 40° for D and ignores a possible H, then for a support reaction of 330 kN there results a Z of approximately 390 kN.

In order to determine these forces, the "truss nodal points" must be correspondingly developed. In particular the tensile force Z over the support must be dealt with through sufficient anchorage. For this purpose the prestressed wires can in certain circumstances be used. From Code SIA 162 [1], Art. 5.05, the transmission or anchorage length should not come to more than 100 wire diameters. In the discussed case this is 70 cm. If a linear stress distribution is assumed over this length, one obtains an anchorage bond stress of about 2.5 N/mm^2.

a)

Truss model

b)

σ_s

linear course

probable course

$l_{ü} \approx 100'\varnothing$

c)

τ_v
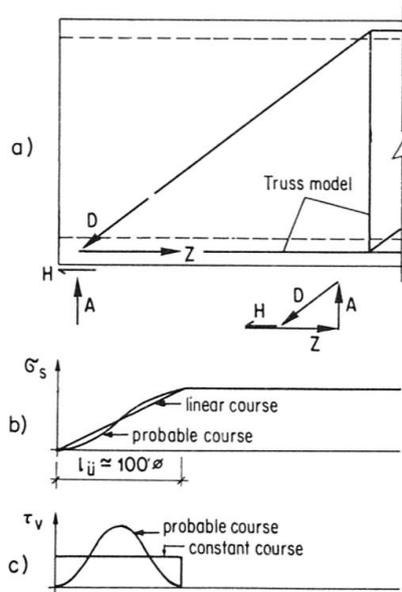
probable course

constant course

Fig. 11 End anchorage:
internal forces following
from the introduction of the
support reaction (truss model);
steel stresses and bond stresses
from the prestress force.

The bond stresses of this size should only
be added into the calculation for the pre-
tensioned wires at the end of the girder
if the following are met:

- slow and simultaneous stressing of all
  wires at a cross-section

- enough spacing between the wires and
  other reinforcement so that concrete may
  be placed and can harden

- sufficient transverse reinforcement to
  take account of the local transverse
  tensile stress.

These stipulations were in the discussed
case either not at all or insufficiently
fulfilled. The wires were cut through one
after another with a power cutter. This
led to a considerable disturbance of bond.
Between various prestressed wires and par-
allel running end stirrups present the
spacing was only a few millimeters. This
did not everywhere keep to the well-known
rule which states that the spacing of the
placed steel should not be less than the
the maximum aggregate diameter (here 15 mm).

And finally, there was only sufficient transverse reinforcement to take up the
local transverse tensile forces in the bottom of the bottom flange, not in the
top.

From these points it can be assumed that the steel stress at the girder end was
under proportional and only further into the girder interior increased to over
proportional (Fig. 11b). Correspondingly the bond stress at the girder end was
relatively small (Fig. 11c). With plausible assumptions the anchorage force comes
within the region of 270 to 380 kN. Here the vertical end stirrups Ø 8 mm lying
parallel to the prestress wires were not investigated. The bond of this steel
with the concrete must, in consequence of the small spacing, have been relative-
ly bad.

Thus also here where the end anchorage alone without transverse bending is con-
sidered, there lies considerable difficulty in showing with computations that un-
der the support reaction prior to collapse an equilibrium condition was possible.

3.5 Action following from Prestressing

In Fig. 12a the layout of the prestress wires in the upper and lower regions is
schematically shown with respect to the introduction of the prestress forces. A
part of these forces goes into the web (transition length S) while the other part
remains in the flanges.

At the transition of the lower flange to the webs there is a calculated shear
stress τ of 4.0 - 6.0 N/mm². This is based upon reasonable assumptions for an
uncracked concrete cross-section. The corresponding considerably smaller main
tensile stress σ_{hz} has the tendency to produce the cracks shown in Fig. 12b.
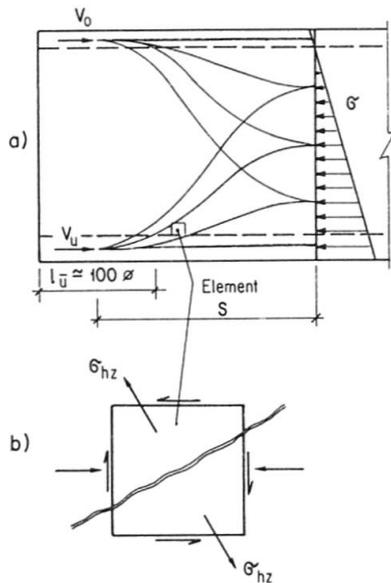
Fig. 12

Stresses and tendency to crack formation in consequence of introduction of pre-stress force.

3.6 Cracks in the other Girders

In Fig. 13 cracks which occured on erected and not erected girders are shown schematically.

Observed only on erected girders   Observed on not erected and erected girders
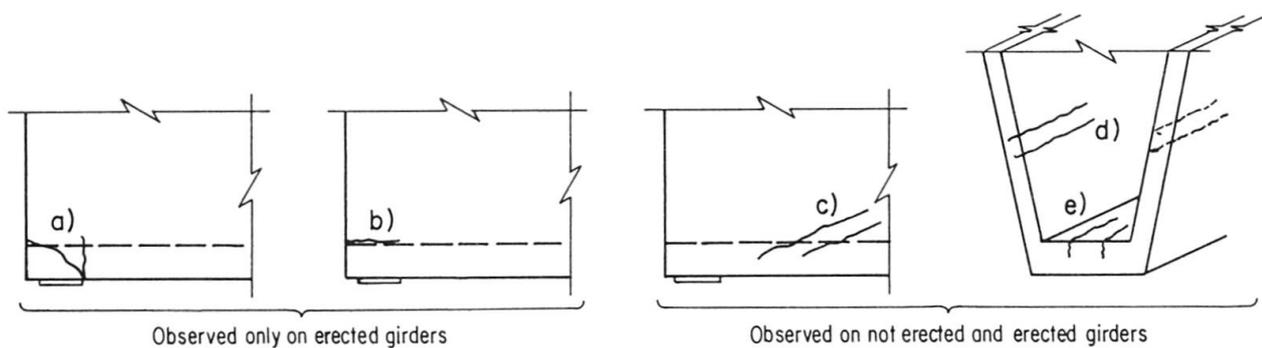
Fig. 13 Crack Types observed on the Girders.

Crack Type a):

These dangerous shearing-off cracks arise from insufficient end anchorage (cf. 3.4).

Crack Type b):

These cracks stem from the effects of transverse bending with concentric and eccentric loads (cf. 3.2 and 3.3).

Crack Type c):

These inclined cracks arise from the spreading of the prestress force from the bottom flange to the web.

Crack Type d): These longitudinal cracks arise probably through shrinkage. There was an adequately strong reinforcement only on the outside of the webs. In addition to this the girders during fabrication were warmed with steam from the inside in order to accelerate hardening.

Crack Type e):

These cracks can be explained in the same way as the Type d) cracks. They were
however considerably wider and longer with the erected girders as with those not
yet erected. This comes from the additional effect of the transverse bending (cf.
3.2 and 3.3).

3.7 Construction Procedure Deficiencies

The inquiry upon the building site and in the fabrication yard showed several
deviations from required procedure. These have already in part been mentioned:

- The tested compressive strength at 28 days of the concrete was approximately
  15% less than the 55 N/mm^2 required by the engineer.

- A maximum edge stress of about 21 N/mm^2 was obtained when prestressing girders
  of round 24 hours age. The tested compressive strength came to round 35 N/mm^2.
  By Code SIA 162 a tested compressive strength of 47 N/mm^2 is required for this
  edge stress.

- The girder were on the average 4 to 5 cm too short. The shortening of the gir-
  der following from prestress, shrinkage and creep was not sufficiently allowed
  for with the formwork measurements. Thus already at erection the possible play
  of the support bolts in the bottom flange holes had to be totally or partially
  used up. The collapsed girder could have only just passed over the bolts.

- With the prestressing of the girders a widely used but nevertheless profession-
  ally incorrect practice was used. The wires were cut through one after another
  with a power cutter. This procedure leads to a strong weakening of the bond be-
  tween single wires and the concrete at the end of the girder. There occurs a
  middle period where the prestress force of the cut wires works upon the beam
  causing it to want to shorten. This causes increased stresses in the not yet
  cut wires and leads to them being drawn out of the girder. This effect has a
  particularly unfavourable result with the last cut wire. Here, in contrast to
  slow and common relaxation of all the wires, there exists an enlarged trans-
  mission or anchorage length.

3.8 Critical Review

The undertaken investigations led to the conclusion that the causes of the fail-
ure of the one girder and also the insufficient carrying capacity of the other
girders were above all based in the design of the support region. The support
bearing points were not placed beneath the webs and there was no end transverse
beam present. The consequently arising transverse bending together with insuffi-
cient end anchorage led to the collapse. The described procedural deficiencies
contributed considerably to an immediate occurrence of collapse. However as the
cause of insufficient strength in the support region these procedural deficiencies
are not significant compared to those of the design. Without the presence of these
procedural deficiencies an insufficient carrying capacity to that required by the
Code would still have been present.

4. REMEDIAL CONSTRUCTION PROPOSALS

A dismantling of the already erected sections did not come into question. Thus, remedial solutions had to be sought that could be practically carried out both in the storage yard and also on the already erected beams. The overall form of the hall could not be altered.

After an extensive study of variants the following remedial proposals were worked through and judged accordingly.

4.1 Steel Shoe

The construction shown in Fig. 14 consists of a steel insert glued onto the inside of the girder and bolted with high tensile bolts through the girder walls and base. This solution was economically favourable. However it had to be put aside due to the large constructional and computational uncertainties (thick resin joints, creep, stress loss of high tensile bolts with a small elongation).
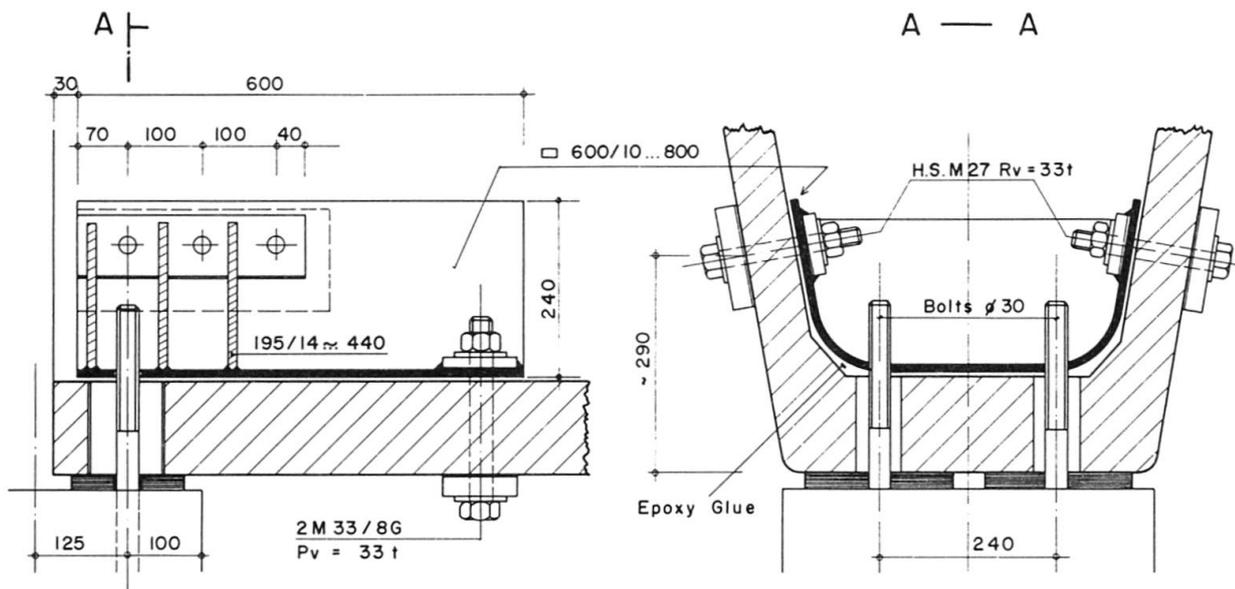

Fig. 14 Remedial solution "Steel Shoe"

4.2 Steel Nib

A further possible remedial solution consists of sitting the girders upon steel nibs as shown in Fig. 15. If the deformation of the support surface is away from the end then sufficient end anchorage is guaranteed. The choice of special small neoprene pads allows the transference of the support forces directly beneath the webs. This largely eliminates the transverse bending. The steel nibs are connected to the in-situ concrete columns with post-tensioned rods.

Two problems stand with this solution. Firstly, there is the introduction of eccentric nib loads to the edge columns. Then secondly there is the taking up of torsional effects on the girder stemming from one sided loading during erection. There are also the further disadvantages of high costs, difficult realisation with the already erected sections and dissatisfactory aesthetics. Therefore this variant was not taken any further.
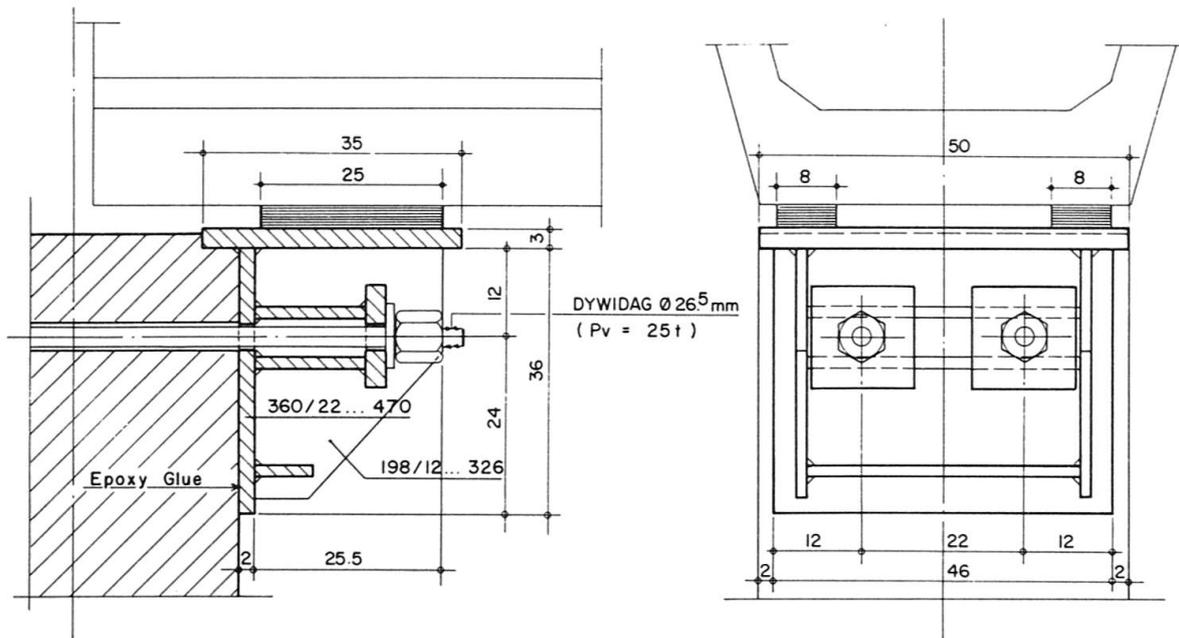
<u>Fig. 15</u>   Remedial solution "Steel Nib"

4.3 Cast-in concrete strengthening at the support

This proposal concerns strengthening of the inside of the box section girder in the support region (see Fig. 17, Variants A + B). The girder end is concentrated for a length of one meter with an internal depth of 37 cm. This new concrete is reinforced, the longitudinal bars having an end plate welded to them which spreads over the adjacent webs and also the bottom flange. The new concrete is connected to the girder with vertical and horizontal prestressed rods.

The structural effect of this strengthening of the girder end can be paraphrased as follows (Fig. 16):
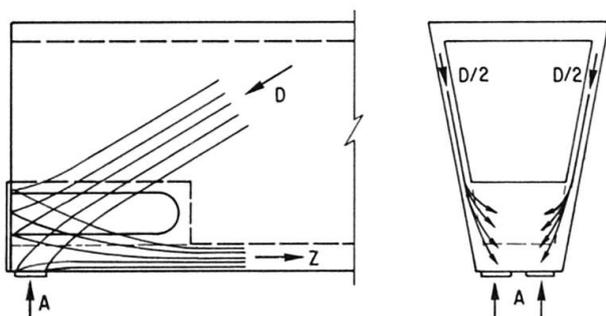

<u>Fig. 16</u>   Flow of the forces with the remedial solution "cast-in concrete strengthening at the support".

The inclined compressive force must be able to be in equilibrium with the support force and the bottom chord tensile force (truss model). The inclined compressive force transfers from the webs through the bond stresses (horizontal prestress) to the new concrete section. It is then taken out through the steel plate and the support. With this the transverse bending of the bottom flange is eliminated or carried by the new concrete section. The horizontal component of the inclined compressive force is taken by the end plate welded to the longitudinal reinforcement into the girder. It is then transferred through bond stresses (vertical prestress) to the existent tension chord.

This solution with its small costs causes the least disturbance on the total form of the hall. Therefore it was decided to take only this solution further.

5. TEST AT THE BUILDING SITE

The intended remedial solution was test loaded at the building site for the following reasons:

1. The proof of an adequate carrying capacity for the strengthened girder could not be brought forward mathematically.

2. In relation to this, the bringing forward of the proof through mathematics involved a longer more complicated solution as could be obtained through the test.

3. The test results offer in comparison to a calculated proof a substantially better basis for a judgement of the safety of the reconstruction.

Thus with the test a sufficiently safe solution could be found that was as advantageous as possible.

5.1 Test girder with two remedial construction variants

The already erected girder 9/E-G was used for the test. In consequence of the blow-like one sided loading at collapse it was damaged lightly through crack formation in the bottom region of the web. This part was carefully repaired with epoxy mortar. This was carried out at end B of the girder.

On the test girder, two different remedial constructions were tried out (Figs. 17 and 18).
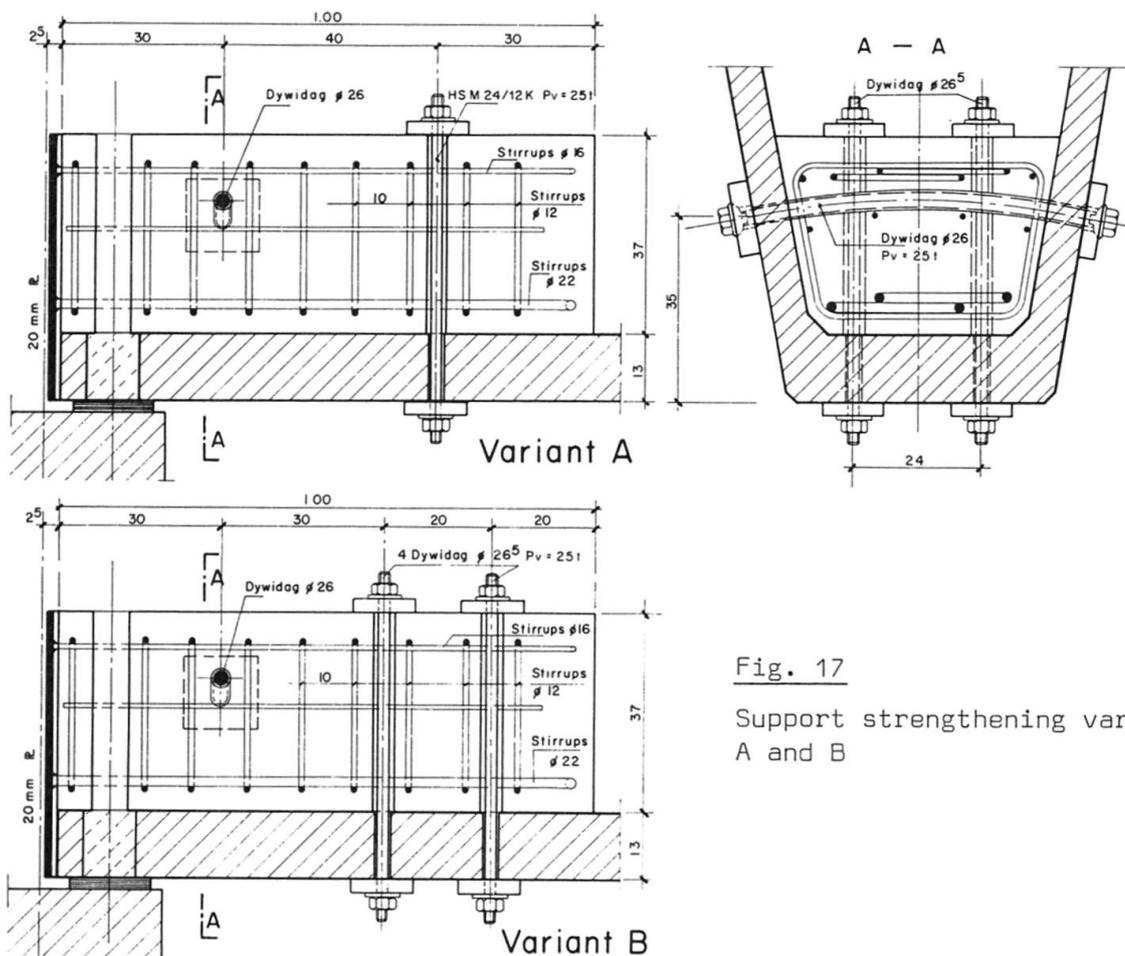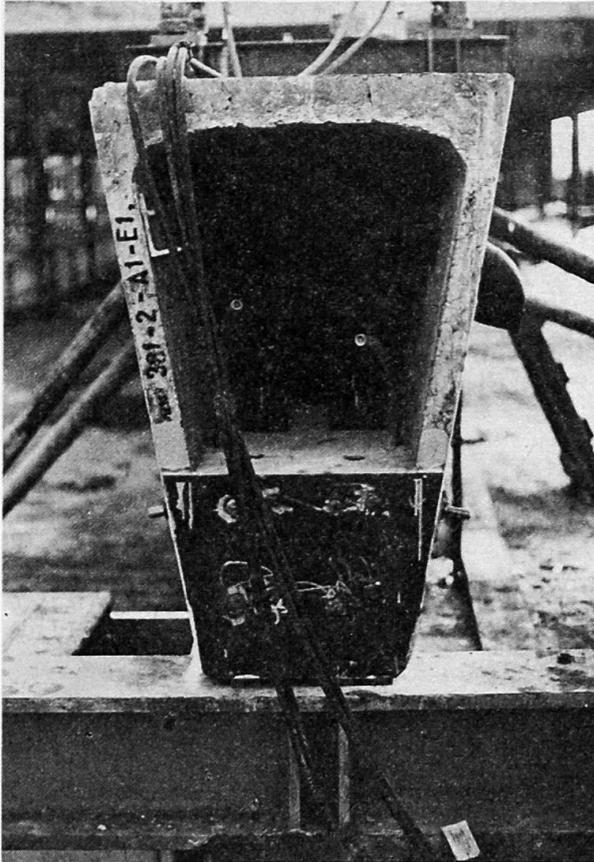

Fig. 17

Support strengthening variants A and B

With variant A the girder end A was
horizontally prestressed with a pre-
liminarily bent Dywidag rod d = 26 mm
and vertically with two high strength
bolts (d = 24 mm). With variant B at
the girder end B, in place of the two
H.S. bolts, Dywidag rods were used.
Using a prestressing jack the prestress
stress forces in the Dywidag rods came
to 250 kN. Through a torque wrench the
prestress in H.S. bolts came likewise
to 250 kN. The Dywidag rods and H.S.
bolts were placed in plastic sheaths
which were not grouted for the test.
The contact surface between the girder
and the concrete infill was scabbled
so that an unevenness of about 3 mm
depth was produced. The concrete of the
new infill had a compressive strength
of about 32 N/mm^2 on the day of the
test. Between the rear vertical end
plate and the webs and also the bottom
flange there was a space of 5 to 10 mm.
This was carefully filled with epoxy
cement. The design of the remedial con-
struction for the girder end is shown
in Fig. 18.

Fig. 18   Girder end, side B of the
          test girder

5.2 Test arrangement

A view of the test arrangement is given in Fig. 19. The girder with 25 m span
was placed over a wall of the very stiff lower storey. The centre point of the
neoprene lay only 7 cm from the girder end which corresponded to the most possible
extreme position in the system.

Since the girder was to be tested only for shear force action (relative to the
support condition) and not for bending the required loading was applied by means
of prestressing jacks only 2.5 m from the support. In order to see the cracks
clearly the girder ends were painted white. The loading was applied at each load
point initially in steps of 100 kN and later of 50 kN.

The jack forces, the deflections at three points, the support settlement and the
crack widths in the webs and bottom flange were all measured at each load step.
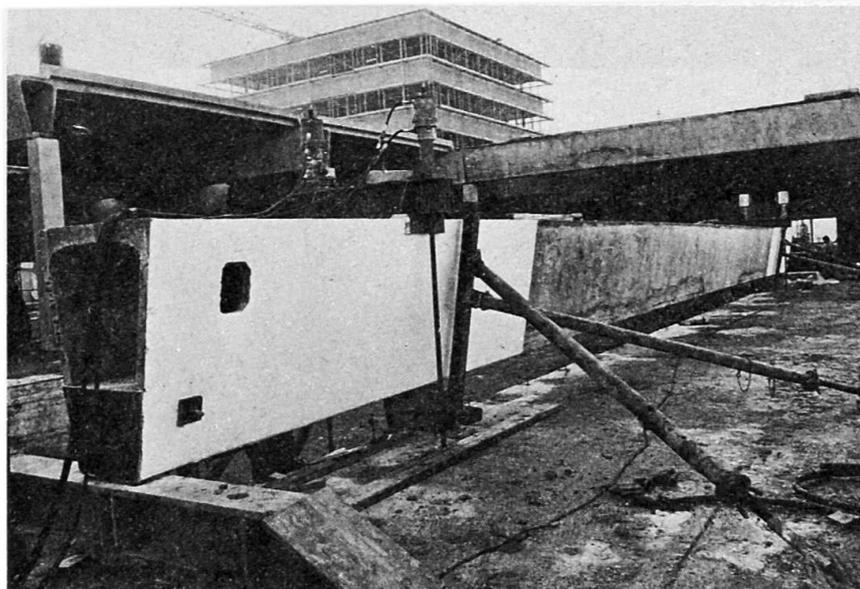The test was carried out on 20th December 1974.

Fig. 19   Test at the building site

5.3 Test Results

Both remedial solutions showed themselves to be workable. The self weight of the
beam and the test fittings led to a support reaction of 115 kN. With a support
reaction corresponding to that of a full working load i.e. with Q(g+p) $\simeq$ 500 kN
there first appeared a few cracks in the web of maximum width 0.15 mm. The mid-
span deflection (without self weight) was 16 mm. Following on from here a load
giving a support reaction corresponding to twice the working load reaction, i.e.
2Q(g+p) $\simeq$ 1000 kN, could be obtained without fracture. Under this maximum loading
the largest crack width in the webs was 0.60 mm and the deflection midspan =
50 mm. Fig.20 shows the midspan deflection and also the crack widths as functions
of the support reaction. Fig. 21 shows the crack pattern at the girder end A.
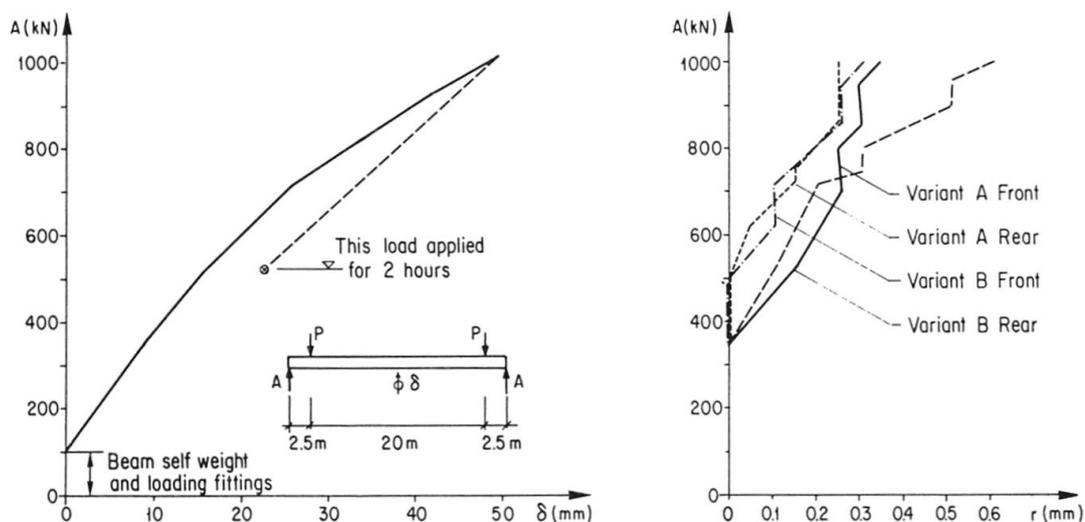

Fig. 20   Midspan deflection of the girder and also the largest
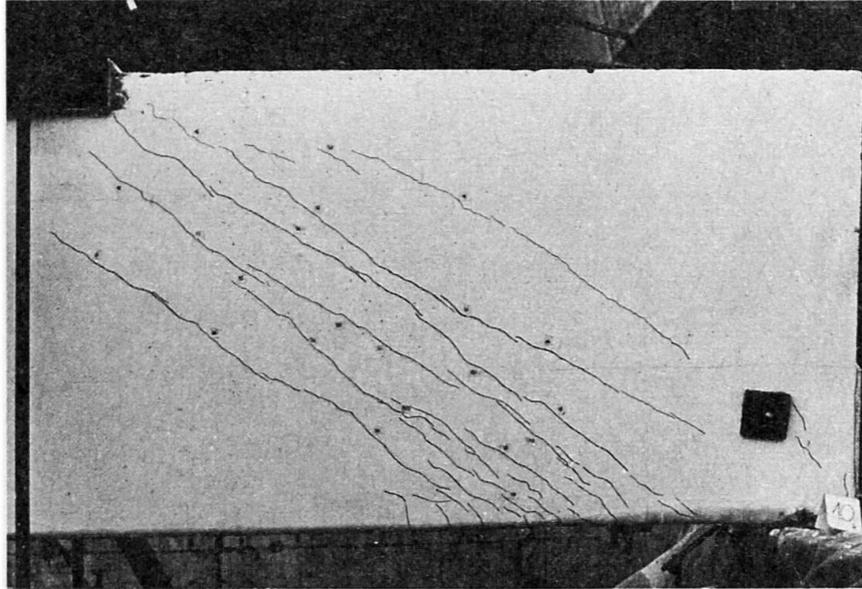   crack width r in the webs as a function of the support reaction

Fig. 21   Crack pattern at girder end A

In the inside of the beam cracks also occurred at the vertical contact surface
between the concrete infill and the webs. They could be observed on the upper
edge of the new concrete section. The cracking began at the girder end with about
1.5Q(g+p). With about 1.9Q(g+p) the contact surface was cracked through right
along the girder. A maximum crack width of 0.4 mm and significant deformations
between the new concrete section and the web could be observed. Between the
variants A and B there was no significant difference. There was no sign of a
failure beginning in the support regions. The girder came through the test ex-
cellently and it was possible to use it again in the building.

6. FINAL RECONSTRUCTION

6.1 Chosen Solution

The observations and results of the building site test formed a very good basis
for the establishing of a final solution. This solution is very similar to the
tested variant A. In place of the H.S. bolts d = 24 mm, material 12 K, two bolts
d = 27 mm and out of the tougher and less susceptible to corrosion material 10 K
were used. The final solution is as in Fig. 22.

The girders A and C show an internal support on grid line A as a result of the
external cantilever. Here there was no problem of end anchorage. The girders
needed only to be strengthened for transverse bending. Shown in Fig. 23, this
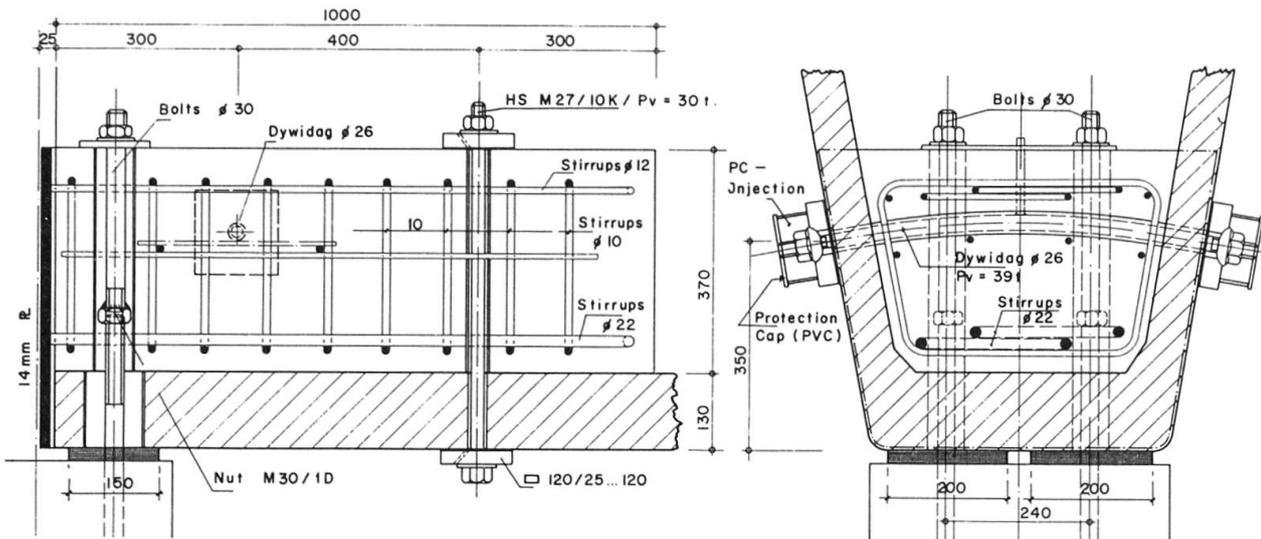solution is close to that for the end support.
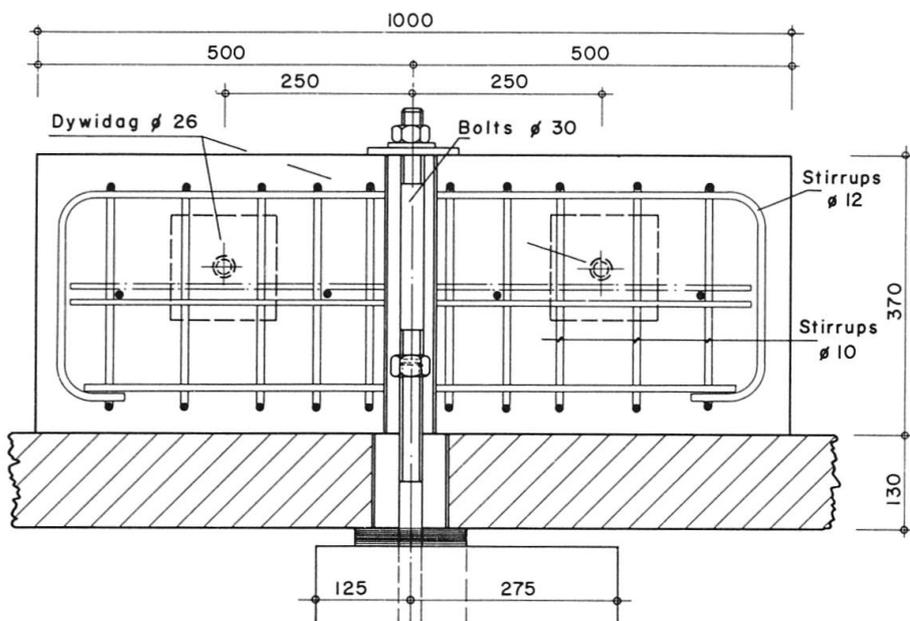
Fig. 22   Adopted remedial solution, end support

Fig. 23   Adopted remedial solution, internal support

6.2 Carrying out of the Reconstruction Work

The load test was carried out before Christmas 1974. The final evaluation and working out of the reconstruction procedure was then subsequently carried out. The reconstruction work began in the middle of February 1975.

The strengthening of the 21 not yet erected girders could be carried out in the storage yard within the month's period. After a further 14 days they could be initially prestressed. This was possible due to the young concrete being cured at elevated temperatures. Since those elements destroyed by the collapse were refabricated in January, the hall could be fully erected in the first half of

April. The remedial work on the already erected girders was quite troublesome.
The new material had to be raised up through the openings between grid lines C
and E into the girders and from there slowly moved by hand to the supports. By
the end of April the members were prestressed and propping placed after the col-
lapse was removed. Finally the grouting of the ducts was carried out (at the
earliest three weeks after the first prestressing). Then the prestress steel was
checked and restressed. The stress loss came to 14%.

Despite the delays through the accident, the building could be handed over to
the management at the timetable change of autumn 1975.

Code safety requirements, service requirements (placement of conduits, accessi-
bility) and aesthetics were all adequately met with this remedial solution.
Fig. 24 shows an interior view of the hall after the successful reconstruction.
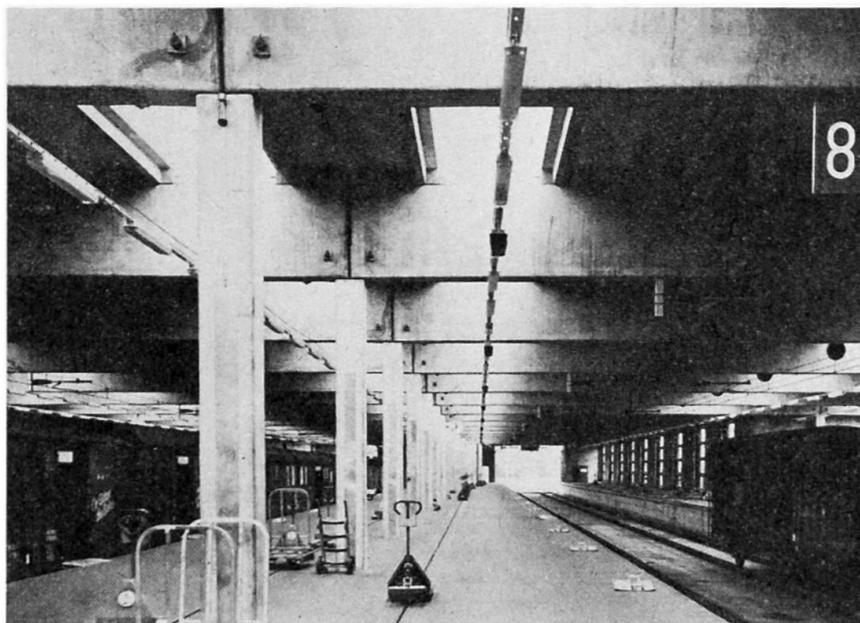

Fig. 24   View of the completed hall

7. RECOMMENDATIONS

From this discussion the following important recommendations can be made:

- Where a box-section girder over a support has no transverse end beam, then it
  is necessary to carefully inspect the support region with respect to trans-
  verse bending and certainly
    . for a possible torsional action on the girder
    . for the shear force action if the support bearing points are not placed
      underneath the webs.

- The end anchorage of a girder is to be designed so that equilibrium between
  the inclined compression force in the web, the support reaction and the tensile
  chord force is possible (truss model). Therefore in the tensile chord the
  reaction force must be anchored behind the theoretical support centre line

with sufficient capacity.

- Allowance for the shortening following from shrinkage and prestress (short and long term) of prefabricated girders must be made in the formwork measurements. This allowance should be based upon careful calculation.

- With the large pretensioned members all the prestress wires should be relaxed simultaneously and slowly. This is to avoid too large transmission lengths.

Reference

[1]   Norm SIA 162 "Berechnung, Konstruktion und Ausführung von Bauwerken aus Beton, Stahlbeton und Spannbeton" (Standard for the Design, Construction and Specification of Concrete Structures, Reinforced and Prestressed). Swiss Engineers and Architects Society, Zürich, 1968.